

Eugene S. Grecheck
Vice President
Nuclear Development

Dominion Energy, Inc. • Dominion Generation
Innsbrook Technical Center
5000 Dominion Boulevard, Glen Allen, VA 23060
Phone: 804-273-2442, Fax: 804-273-3903
E-mail: Eugene.Grecheck@dom.com



November 4, 2009

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D. C. 20555

Serial No. NA3-09-033R
Docket No. 52-017
COL/BCB

DOMINION VIRGINIA POWER
NORTH ANNA UNIT 3 COMBINED LICENSE APPLICATION
RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION LETTER NO. 042
(FSAR CHAPTER 2 AND PART 10)

On September 18, 2009, the NRC requested additional information to support the review of certain portions of the North Anna Unit 3 Combined License Application (COLA). The letter contained two RAIs. The responses to the RAIs are provided in Enclosures 1 and 2:

- RAI Question 02.05.04-20 Backfill Placement, Testing and ITAAC
- RAI Question 02.05.04-21 Engineering Properties of Concrete Fill

The information provided in the RAI responses will be incorporated into a future submission of the North Anna Unit 3 COLA, as described in the Enclosures.

Please contact Regina Borsh at (804) 273-2247 (regina.borsh@dom.com) if you have questions.

Very truly yours,

Eugene S. Grecheck

DD89
HRO

COMMONWEALTH OF VIRGINIA

COUNTY OF HENRICO

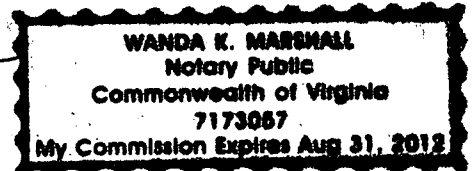
The foregoing document was acknowledged before me, in and for the County and Commonwealth aforesaid, today by Eugene S. Grecheck, who is Vice President-Nuclear Development of Virginia Electric and Power Company (Dominion Virginia Power). He has affirmed before me that he is duly authorized to execute and file the foregoing document on behalf of the Company, and that the statements in the document are true to the best of his knowledge and belief.

Acknowledged before me this 4th day of November 2009

My registration number is 7173057 and my

Commission expires: August 31, 2012

Wanda K. Marshall
Notary Public



Enclosures:

1. Response to NRC RAI Letter No. 042, RAI Question No. 02.05.04-20
2. Response to NRC RAI Letter No. 042, RAI Question No. 02.05.04-21

Commitments made by this letter:

1. The information provided in the RAI responses will be incorporated into a future submission of the North Anna Unit 3 COLA, as described in the Enclosures.

cc: U. S. Nuclear Regulatory Commission, Region II
T. A. Kevern, NRC
J. Jessie, NRC
J. T. Reece, NRC

ENCLOSURE 1

Response to NRC RAI Letter 042

RAI Question 02.05.04-20

NRC RAI 02.05.04-20

RAI 02.05.04-13 addressed the backfill ITAAC. The staff requests additional information as follows:

In response to RAI 02.05.04-13, detailed information was provided on confirmatory field testing of seismic Category I structural backfill and associated ITAAC. For the field density test, you stated that "a minimum of one test will be performed per lift with at least one test made for every 10,000 ft² of fill placed." You also revised ITAAC for backfill under Category I structures and included wording that would permit modification of the shear wave velocity (SWV) criteria through site-specific analysis. As a follow up to this response, a teleconference was held on September 10, 2009 with the applicant and NRC staff. Please provide the following:

- (a) Justify whether one test for every 10,000 ft² for a backfill field density test is adequate without mentioning the thickness of the backfill lift. In addition, consider the following field density test frequency guidance as provided in some commonly used standards: (1) no lift should be more than 8 inches in thickness and (2) a routine acceptance control test should be conducted for, at least, every 200 cubic yards of compacted backfill material in critical areas.*
- (b) Revise the backfill ITAAC wording considering the NRC's August 7, 2009 letter to NEI regarding the NRC staff's position and standard wording for backfill ITAAC under Category I structures ("Response to the Nuclear Energy Institute on Backfill Inspection, Test, Analysis and Acceptance Criteria" ML0920905970).*

Dominion Response

- (a) The response to RAI 02.05.04-13 indicated that structural fill will be compacted in thin lifts with loose lift thickness no greater than 12 in., and that a minimum of one density test will be performed for every 10,000 ft² of fill placed.*

Table 5.6 of ASME NQA-1-1994 (ASME 1994) provides a list of various field density testing frequencies depending on the circumstances of the fill placement. The most stringent requirement listed is one field test every 200 to 300 cubic yards of fill placed. To better align with industry standards, the FSAR will be revised to state that compacted structural fill placement and testing will follow the guidelines of ASME NQA-1-1994, and that at least one field density test will be performed per lift and for no more than every 250 cubic yards of fill placed.

- (b) COLA Part 10, "Tier 1 ITAAC," will be revised to address the NRC's August 7, 2009, letter to NEI on standard wording for backfill ITAAC under Category I structures.

Reference

ASME (1994), American Society of Mechanical Engineers, ASME NQA-1-1994, Quality Assurance Program Requirements for Nuclear Facility Applications, New York, NY, 1994.

Proposed COLA Revision

FSAR Section 2.5.4.5.3 and COLA Part 10, "Tier 1 ITAAC," will be revised as shown in the attached markups.

Revision 6 of the ESBWR DCD changed the shear wave velocity (SWV) soil site parameter from equivalent uniform SWV to minimum SWV. As a result of this change, the North Anna Unit 3 site value for SWV no longer falls within the DCD site parameter for the structural fill under the Fire Water Service Complex. To address this condition, a future COLA submittal will revise COLA Part 7, Report on Departures, to include a request for exemption from Tier 1 and identify a departure from Tier 2 (consistent with NRC's August 7, 2009 letter to NEI on backfill ITAAC). These COLA Part 7 and associated FSAR changes will be implemented following the completion of the site-specific soil-structure interaction (SSI) analysis, as described in Dominion's response to RAI 02.05.04-13.

Markup of North Anna COLA

The attached markup represents Dominion's good faith effort to show how the COLA will be revised in a future COLA submittal in response to the subject RAI. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be somewhat different than as presented herein.

2.5.4.5.3 Structural Fill Sources, Compaction and Quality Control

Although a large amount of Zone IIA soil will be excavated for Unit 3, this material will not be used as structural fill to support Seismic Category I or II structures.

Structural fill is either lean concrete or a sound, well-graded granular material. The anticipated extent of the concrete and granular fill is shown on the foundation cross-sections on Figure 2.5-229 through Figure 2.5-234. The concrete fill is used to replace any moderately to severely weathered rock (Zone III) exposed at the bottom of the excavations for the Seismic Category I RB/FB and Control Building foundation mats. The concrete fill will be designed to result in a shear wave velocity in the same range as that of the Zone III-IV rock.

The granular structural fill material does not exist naturally on site. However, given the large amount of rock that will need to be excavated for Unit 3, it will be economical to set up a crushing and blending plant onsite to produce crushed aggregate to the required gradation specifications for use as structural fill. The rock will be crushed down to well-graded, angular or sub-angular sand and gravel-sized particles conforming to the gradation of Size No. 21A specified by the Virginia Department of Transportation (DOT) Road and Bridge Specifications (SSAR Reference 166). This gradation is shown in Figure 2.5-277. The soundness of the aggregate will be confirmed using sulfate soundness and Los Angeles abrasion tests. This structural fill will be placed in lifts not exceeding 12 inches loose thickness. The structural fill is compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557 (SSAR Reference 165) as stated in the ITAAC for backfill compaction in COLA Part 10, and to within 3 percent of its optimum moisture content. Compaction will be performed with a heavy steel-drummed vibratory roller, except within 1.5 m (5 ft) of a structure wall, where smaller compaction equipment will be used in conjunction with reduced lift thickness to minimize excess pressures against the wall. As noted in Section 2.5.4.2.5.b, based on the type of material and its degree of compaction, $N_{60} = 50$ blows/0.3 m (1 ft) and $\phi' = 40$ degrees were assumed as reasonable and conservative for this structural fill.

Although proposed structural backfill material from the site is not presently available, bulk samples of similar material will be obtained from a quarry in the site vicinity that crushes the native rock (sound gneiss or

schist) to the VDOT Size 21A gradation. Laboratory tests will be used to confirm the properties of the structural backfill, and will include:

- Confirmatory gradation tests
- Modified Proctor compaction tests to provide values of maximum density and optimum moisture content
- Consolidated-undrained (CU) triaxial compression tests, with porepressure measurements, on compacted samples at different confining pressures to verify the angle of internal friction
- RCTS testing

Since the gradation of the fill material falls within a relatively narrow band, the test results should be consistent, and so the number of laboratory tests can be limited. Two each of the modified Proctor, CU triaxial, and RCTS tests should provide sufficient data. These tests support the site-specific soil-structure interaction (SSI) analysis in Section 3.7.

As an alternative or supplement to the onsite crushed rock, dense-graded aggregate can be used as structural fill material. Dense-graded aggregate will conform to Virginia DOT Size 21A (SSAR Reference 166) as noted in the previous paragraph.

Fill placement and compaction control procedures will be addressed in a technical specification that includes requirements for suitable fill, sufficient testing to address potential material variations, and in-place density testing frequency, ~~i.e., a minimum of one test per 930 m² (10,000 ft²) of fill placed.~~ Compacted structural fill placement and testing will follow the guidelines of ASME NQA-1 (Reference 2.5-221). At least one field density test will be performed per lift and for no more than every 191 m³ (250 yd³) of fill placed. ~~†~~ The technical specification also includes requirements for an on-site testing laboratory for quality control (gradation, moisture-density, placement, compaction, etc.) and requirements to ensure that the fill operations conform to the earthwork specification. The soil testing firm is required to be independent of the earthwork contractor and to have an approved quality assurance program. Sufficient laboratory compaction (modified Proctor) and grain size distribution tests will be performed to ensure that variations in the fill material are accounted for. (Variations in the crushed and blended rock are expected to be minimal.)

- 2.5-213 Imai, T. and K. Tonouchi, "Correlation of N-Value with S-Wave Velocity and Shear Modulus," Proceedings, Second European Symposium on Penetration Testing, No. 1, Balkema, Amsterdam, 1982.
- 2.5-214 U.S. NRC. RG 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Plants," November 2003.
- 2.5-215 American Concrete Institute. "Code Requirements for Nuclear Safety-Related Structures," ACI 349-01, 2001.
- 2.5-216 Poulos, H.G., and E. H. Davis, Elastic Solutions for Soil and Rock Mechanics, John Wiley and Sons, Inc., New York, 1974.
- 2.5-217 Seed, H. B., and R. V. Whitman, "Design of Earth Retaining Structures for Dynamic Loads," Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, ASCE, New York, 1970.
- 2.5-218 Ostadan, F. and W. H. White, "Lateral Seismic Soil Pressure: an Updated Approach," Proceedings of U.S.-Japan SSI Workshop, Menlo Park, CA, 1998.
- 2.5-219 Geo-Slope International Ltd. SLOPE/W Version 6.13, Calgary, Alberta, Canada, 2004.
- 2.5-220 ASTM International, ASTM D 1587-00, Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes, West Conshohocken, PA, 2000.
- 2.5-221 ASME NQA-1, Table 5.6, "Quality Assurance Program Requirements for Nuclear Facilities," 1994.

2.4 Site-Specific ITAAC

The Site Specific ITAAC are provided in the following sections. Site specific systems were evaluated against selection criteria in Section 14.3. If a site-specific system described in the FSAR does not meet an ITAAC selection criterion, only the system name and the statement "No entry for this system" is provided.

2.4.1 ITAAC for Seismic Category I Backfill Under Category 1 Structures the FWSC Foundation

Design Description

~~Backfill under Category I structures is installed up from competent bearing layer to meet average and minimum soil density requirements specified in Table 2.4.1-1.~~

The Seismic Category I backfill material under the FWSC foundation is installed to meet a minimum of 95 percent Modified Proctor density.

The shear wave velocity of Seismic Category I backfill material is greater than or equal to 449 ft/sec at the depth of the FWSC foundation and below.

Inspections, Tests, Analyses and Acceptance Criteria

~~Table 2.4.1-2~~ Table 2.4.1-1 provides a definition of the inspections, tests and/or analyses, together with associated acceptance criteria, for the backfill under ~~Category I structures~~ ITAAC the FWSC foundation.

~~Table 2.4.1-1 Compaction Requirements for Backfill Under Category I Structures~~

Average Compaction (all tests)	97% Compaction
Number of compaction test results < 95% compaction	10% of test results
Number of compaction test results < 93% compaction	zero

Table 2.4.1-1 ~~Table 2.4.1-2~~ **ITAAC For Seismic Category I Backfill Material Under Category 1 Structures the FWSC Foundation**

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1. Backfill under Category I structures is installed to meet average and minimum soil density requirements specified in Table 2.4.1-1 <u>Seismic Category I backfill material under the FWSC foundation is installed to meet a minimum of 95% of the Modified Proctor density.</u>	4. Inspection and <u>Testing will be performed during placement of the backfill materials.</u>	4. A report exists that concludes that the installed backfill material under Category I structures meets the average and minimum soil density requirements specified in Table 2.4.1-1 <u>the FWSC foundation meets a minimum of 95% of the Modified Proctor density.</u>
2. <u>Shear Wave Velocity of Seismic Category I backfill material is greater than or equal to 449 ft/sec at the depth of the FWSC foundation and below.</u>	<u>Field measurements and analyses of shear wave velocity in backfill will be performed when backfill placement is at the elevation of the bottom of the FWSC foundation and at finish grade.</u>	<u>An engineering report exists that concludes that the shear wave velocity within the backfill material placed under the FWSC at the foundation depth and below is greater than or equal to 449 ft/sec.</u>

ENCLOSURE 2

Response to NRC RAI Letter 042

RAI Question 02.05.04-21

NRC RAI 02.05.04-21

RAI 02.05.04-12 addressed concrete fill. The staff requests additional information as follows:

In response to RAI 02.05.04-12, you provided a detailed description of properties of concrete fill and specified that it will have a shear wave velocity of at least 6,295 ft/sec and a strength of 2,500 psi. Your response also indicated that there would be no COLA revision regarding this issue.

As a follow up to this response, a teleconference was held on September 10, 2009 with the applicant and NRC staff. The staff inquired as to why the concrete fill properties had not been included in the FSAR. The FSAR states that concrete fill will be placed under Category 1 structures; however, it does not provide a description of the concrete fill properties. Please provide a clear description of concrete fill properties in the FSAR or justify why such description is not needed.

Dominion Response

The FSAR will be revised to provide a description of the engineering properties of concrete fill, such as minimum strength, unit weight, Poisson's ratio, and minimum shear wave velocity.

Proposed COLA Revision

FSAR 2.5.4.2.5 and Table 2.5-212 will be revised as shown in the attached markups.

Markup of North Anna COLA

The attached markup represents Dominion's good faith effort to show how the COLA will be revised in a future COLA submittal in response to the subject RAI. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be somewhat different than as presented herein.

program with work procedures developed specifically for the Unit 3 project. Soil and rock samples were shipped under chain-of-custody protection from the storage area (described in Section 2.5.4.2.3) to the testing laboratory. When required, samples sent to the testing laboratory were divided and/or shipped to an appropriate testing laboratory under chain-of-custody rules. Laboratory testing of soil and rock samples, except for chemical tests and resonant column torsional shear (RCTS) tests, was performed at the MACTEC laboratories in Charlotte and Raleigh, North Carolina and Atlanta, Georgia. Chemical testing for pH, sulfates and chlorides in selected soil samples was conducted by Severn Trent Laboratories in Earth City, Missouri. RCTS testing of selected soil samples was performed by Fugro Inc. in Houston, Texas, under the technical direction of Dr. K. H. Stokoe of the University of Texas in Austin.

Since the Unit 3 power block area is approximately 460 m (1500 ft) southwest of the center of the Unit 2 Containment Building, the tests focused on verifying that the properties of the soil and rock beneath the Unit 3 power block area were similar to those beneath Units 1 and 2 as determined during previous studies. In addition, chemical tests (for corrosiveness toward buried steel and aggressiveness toward buried concrete) and RCTS tests (for shear modulus and damping ratio variation with cyclic strain) were run on selected saprolite samples.

The details and results of the laboratory testing are included in Appendix 2.5.4AA, except for the RCTS test results which are included in Appendix 2.5.4AAS1. Appendix 2.5.4AA includes references to the industry standards used for each specific laboratory test. The results of the tests on soil samples (excluding strength and RCTS tests) are summarized in Table 2.5-210. Table 2.5-211 gives the results of the unconfined compression tests on the rock cores. The results of the RCTS tests are shown in Figure 2.5-223.

The results of the laboratory tests as they relate to the engineering properties of the soil and rock are described in Section 2.5.4.2.5.

2.5.4.2.5 Engineering Properties

The engineering properties for Zones IIA, IIB, III, III-IV, and IV derived from the Unit 3 field exploration and laboratory testing programs are provided in Table 2.5-212 and described in the following paragraphs. These engineering properties are similar to those obtained from the previous field and laboratory testing programs (as shown in

SSAR Table 2.5-45), with some differences. Where there are differences, the impact from an engineering standpoint is usually either the same or more favorable.

The following paragraphs discuss selected properties shown in Table 2.5-212 under the subheadings: a) rock properties, including concrete fill; b) soil properties, including structural fill; c) RCTS results; and d) chemical properties.

a. **Rock and Concrete Fill Properties**

Rock

In general, the rock strength and stiffness values, derived from the field and laboratory testing of the Unit 3 rock, are higher than given in the SSAR. This could reflect less fractured or weathered rock beneath the Unit 3 area, and/or better rock coring equipment and techniques that produced better quality cores.

The Recovery and RQD are based on the results presented for each core in the boring logs in Appendix 2.5.4AA. The RQDs from the borings for Strata III, III-IV and IV are plotted versus elevation in Figure 2.5-224. For Stratum III, RQD generally ranges from zero to around 50 percent, with some higher values. The average value is about 20 percent. For Stratum III-IV, RQD generally ranges from around 50 to 90 percent. The average value is about 65 percent (compared to 50 percent in the SSAR). For Stratum IV, RQD is generally above 80 percent and mostly above 90 percent. The average value is about 95 percent. The average recovery values for Zone III, III-IV and IV are 55 percent, 90 percent, and 98 percent, respectively.

The unconfined compressive strengths and unit weights in Table 2.5-212 are based on the rock strength test results shown in Table 2.5-211. The elastic modulus values are also based on the values shown in Table 2.5-211. The shear modulus values are derived from the elastic modulus values using the Poisson's ratio values tabulated in Table 2.5-212. These higher strain shear modulus values agree well with the low strain values derived from the geophysical tests performed for the Unit 3 exploration program described in Section 2.5.4.4. These high and low strain shear modulus values are essentially the same for high strength rock, certainly for the Zone IV and Zone III-IV rock. Some strain softening has been allowed in the case of the Zone III rock, as described in Section 2.5.4.7. Low strain is defined here as 10^{-4} percent while high

strain is taken as 0.25 to 0.5 percent, the amount of strain frequently associated with settlement of structures on soil.

The shear and compression wave velocities in Table 2.5-212 are based on suspension P-S velocity logging performed as part of the Unit 3 exploration program (Appendix 2.5.4AA). These results are summarized in Section 2.5.4.4.4.

Concrete Fill

As stated in Section 2.5.4.10, if Zone III weathered rock or fractured rock is encountered at foundation subgrade level of the RB/FB, it will be removed and replaced with concrete fill. The concrete fill will have a minimum strength of 2,500 psi, with a unit weight and Poisson's ratio of 145 pcf and 0.15, respectively. The bearing capacity of concrete fill is addressed in Section 2.5.4.10.1.

Figures 2.5-229 through 2.5-232 show fractured or weathered rock will be removed from up to 22 ft depth below the base of the RB/FB foundation. Analysis indicates that if the top 25 ft of rock beneath the RB/FB foundation is replaced with concrete, the seismic response at foundation level decreases with increasing shear wave velocity (V_s) of the concrete. Based on the calculated Selected Mean V_s values at and below the RB/FB foundation (shown in Figure 2.5-241), the Selected Median V_s of the in-situ rock at 25 ft below the RB/FB foundation base is approximately 5,825 ft/sec. Therefore, the V_s of the concrete fill should be equal to or greater than 5,825 ft/sec to ensure that the seismic response of the column that includes the concrete fill is equal to or less than the response from the original analysis of the in-situ rock. Further analysis indicates that concrete with strength of 2,500 psi has a V_s of at least 6,295 ft/sec.

b. Soil Properties

Zone IIA Saprolite

Grain size curves from sieve analyses of Zone IIA silty and clayey sand, and sandy silt samples are shown in Appendix 2.5.4AA. The tests were run mainly on the silty sand samples with more than 90 percent having fines contents of less than 50 percent. Figure 2.5-225 shows fines content versus depth from these tests. The median fines content for the Zone IIA saprolite is about 25 percent, with the majority of samples having a Unified Soil Classification System (USCS) classification (Reference 2.5-209) of SM.

NAPS COL 2.0-29-A **Table 2.5-212 Engineering Properties for Soil and Bedrock of Subsurface Materials**

Stratum	Structural Fill	Concrete Fill	Zone IIA	Zone IIB	Zone III	Zone III-IV	Zone IV
Description	Gravelly materials derived from crushing rock material		Saprolite – core stone less than 10% of volume of overall mass	Saprolite – core stone 10% to 50% of volume of overall mass	Weathered rock – core stone more than 50% of volume of overall mass	Moderately weathered to slightly weathered rock	Parent rock – slightly weathered to fresh rock
USCS symbol	GW	=	SM, SC	SM	-	-	-
Total unit weight, g (pcf)	130	<u>145</u>	125	130	150	163	164
Fines Content (%)	6-12	=	25	20	-	-	-
Natural water content, w_N (%)	-	=	19	14	-	-	-
Atterberg limits		=					
Liquid limit, LL	-	=	-	-	-	-	-
Plastic limit, PL	-	=	-	-	-	-	-
Plasticity index, PI	-	=	-	-	-	-	-
Measured SPT N-value (blows/ft)	-	=	15	75	Ref	-	-
Adjusted SPT N_{60} -value (blows/ft)	50	=	20	100	Ref	-	-
Undrained properties							
Undrained shear strength, s_u (ksf)	-	=	-	-	-	-	-
Unconfined compressive strength, q_u (ksi)	-	<u>2.5</u>	-	-	1.0	9.0	17.0

NAPS COL 2.0-29-A **Table 2.5-212 Engineering Properties for Soil and Bedrock of Subsurface Materials**

Stratum	Structural Fill	Concrete Fill	Zone IIA	Zone IIB	Zone III	Zone III-IV	Zone IV
Description	Gravelly materials derived from crushing rock material		Saprolite – core stone less than 10% of volume of overall mass	Saprolite – core stone 10% to 50% of volume of overall mass	Weathered rock – core stone more than 50% of volume of overall mass	Moderately weathered to slightly weathered rock	Parent rock – slightly weathered to fresh rock
Drained properties							
Effective cohesion, c' (ksf)	0	=	0.125	0	-	-	-
Effective friction angle, ϕ' (degrees)	40	=	33	40	-	-	-
Shear wave velocity, V_s (ft/sec)	1,100	<u>6,295</u>	850	1,600	3,000	4,500	9,000
Compression wave velocity, V_p (ft/sec)	2,400	<u>9,810</u>	1,800	3,500	7,300	9,000	16,000
Poisson's ratio, μ (high strain)	0.3	<u>0.15</u>	0.35	0.3	0.4	0.33	0.27
Poisson's ratio, μ (low strain)	0.37	<u>0.15</u>	0.35	0.37	0.4	0.33	0.27
Elastic modulus (high strain), E_h	1,800 ksf	<u>2,850 ksi</u>	720 ksf	3,600 ksf	400 ksi	1,900 ksi	7,250 ksi
Elastic modulus (low strain), E_l	13,000 ksf	<u>2,850 ksi</u>	7,500 ksf	28,000 ksf	800 ksi	1,900 ksi	7,250 ksi
Shear modulus (high strain), G_h	700 ksf	<u>1,240 ksi</u>	270 ksf	1,400 ksf	150 ksi	700 ksi	2,900 ksi
Shear modulus (low strain), G_l	5,000 ksf	<u>1,240 ksi</u>	2,800 ksf	10,000 ksf	300 ksi	700 ksi	2,900 ksi
Consolidation characteristics							
Compression ratio, CR		=			-	-	-
Recompression ratio, RR		=			-	-	-

NAPS COL 2.0-29-A **Table 2.5-212 Engineering Properties for Soil and Bedrock of Subsurface Materials**

Stratum	Structural Fill	Concrete Fill	Zone IIA	Zone IIB	Zone III	Zone III-IV	Zone IV
Description	Gravelly materials derived from crushing rock material		Saprolite – core stone less than 10% of volume of overall mass	Saprolite – core stone 10% to 50% of volume of overall mass	Weathered rock – core stone more than 50% of volume of overall mass	Moderately weathered to slightly weathered rock	Parent rock – slightly weathered to fresh rock
Coefficient of subgrade reaction, k_1 (kcf)	2,000	=	260	2,000	-	-	-
Coefficient of sliding	0.55	<u>0.7</u>	0.35	0.45	0.6	0.65	0.7
Static earth pressure coefficients							
Active, K_a	0.22	=	0.30	0.22	-	-	-
Passive, K_p	4.60	=	3.40	4.60	-	-	-
At-rest, K_0	0.36	=	0.50	0.36	-	-	-
Optimum moisture content, w_{opt} (%)	=	=	14	=	-	-	-
Maximum dry unit weight, g_{max} (pcf)	=	=	116	=	-	-	-
Rock Quality Designation, RQD (%)	-	=	-	-	20	65	95